VOLUME 81 SEPARATE No. 658

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

MARCH, 1955

DISCUSSION OF PROCEEDINGS - SEPARATES

215, 311, 513, 514, 516

SOIL MECHANICS AND FOUNDATIONS DIVISION

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Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

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This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N.Y.

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Discussion of
"ESTIMATING THE TRUE CONSOLIDATION BEHAVIOR
OF CLAY FROM LABORATORY TEST RESULTS"

by John H. Schmertmann
(Proc. Sep. 311)

JOHN H. SCHMERTMANN,[1] J.M. ASCE—Mr. Crisp questions the validity of the parallel relationship between laboratory recompression and rebound slopes which the writer used as a working approximation. The writer agrees with Mr. Crisp's opinion that the rebound slope varies with the physical and mineralogical structure of the clay and the void ratio (function of consolidation pressure) from which the rebound begins. However, an attempt will now be made to show that this variation is somewhat predictable and recompression slope magnitudes[2] can be estimated without the extreme care Mr. Crisp believes may be necessary.

Some preliminary work indicates that for many clays the change in rebound slope magnitude with decreasing void ratio may be primarily a function of the rebound slope (C_r) and the void ratio at the start of the rebound (e_r). The following tentative equation expresses this relationship:

$$\log\left(\frac{C_{r_1}}{C_{r_2}}\right) = 2.5 \log\left(\frac{e_{r_2} + 1}{e_{r_1} + 1}\right)$$

The 2.5 constant is an average value which subsequent work may show to have a predictable variation with clay structure—perhaps as expressed by limits. Laboratory rebound and recompression cycles show that the recompression slope (C_x) can be approximated by 1.15 C_r. With the above background work it was believed that the best method of plotting available data on comparisons between laboratory recompression and rebound slopes would be a log plot of $\left(\frac{e_r + 1}{e_x + 1}\right)$ vs. C_x/C_r; where e_x = void ratio at the start of the rebound which preceded the recompression whose slope = C_x; and e_r and C_r refer to the final rebound of the laboratory consolidation test. Such a plot is presented on Figure 16.

The data on Fig. 16 were obtained from 59 individual consolidation tests on preconsolidated clays showing distinct recompression slopes. Fig. 16 includes only comparisons from samples showing a low degree of sample disturbance. As shown by the legend, symbol type denotes geologic origin and symbol size denotes a qualitative estimate of sample disturbance. In geographic scope these data include 49 tests on U. S. clays from 16 sites in 9 states; and 10 tests from 5 sites in 4 foreign countries. In 9 of the tests an intermediate virgin rebound-reload cycle was performed and thus these tests yielded more than one comparison point.

1. Corporal, Corps of Engrs., U. S. Dept. of the Army, Snow, Ice and Permafrost Research Est., Wilmette, Ill.
2. All slope magnitudes measured by Δ e over one pressure cycle on conventional semi-log p-e plot.

The scatter of the data on Fig. 16 was expected. The magnitude of the recompression slope, C_x, is significantly increased by sample disturbance. Since disturbance is known to be variable one should expect scatter. In addition, perameters other than void ratio may be important.

Using the following system of weighing the individual results according to sample disturbance: minimum - 5, slight - 3, some - 1, the following conclusions are drawn from the data presented:

The heavy solid line on Fig. 16, whose slope corresponds to the 2.5 constant of the previous equation, indicates the most probable variation of recompression slope with void ratio. The light solid lines indicate the ±15 % zone of error about the heavier line. Based on the data presented, one may have 65 % confidence that if one uses the relationship expressed by the heavy line the error in C_x will not be greater than 15 %. The data points within the dashed lines indicate that one may have 80 % confidence that the error will not be greater than 25 %. The latter is within the practical accuracy of settlement analyses. The maximum error is about 45 %. It is therefore believed that the heavy line on Figure 16 represents a usable empirical relationship.

When using Fig. 16 one must remember that e_x represents the void ratio from which the field rebound started. Unlike e_r, e_x is not definitely known and must be estimated from the pressure-void ratio diagram. As examples, the writer chose the following e_x values for the tests presented in Fig. 6: (a) 0.77, (b) 1.06, (c) 1.53, 1.15, (d) 0.73. These e_x values were easy to estimate because these tests show a minimum of sample disturbance and the recompression slopes are distinct. In more difficult cases a short graphical successive approximation procedure, working back from the in situ pressure (lower pressure if swelling occurred) and void ratio will be helpful.

It is interesting to note on Fig. 16 that, for the 9 tests presented, a cyclic rebound and recompression procedure, in which $e_x = e_r$, yields an average C_x/C_r value of 1.15, with a range of ± .10. Thus, a laboratory rebound slope is about 87 % of the average slope of the subsequent recompression. The reason for the steeper recompression slope can be seen by comparing C_x and C_r values on Figure 17. These laboratory test results were obtained from two carefully cut test pit samples.

The information in Figures 16 and 17 make it possible to recommend the following changes in two of the steps previously recommended in the reconstruction of the true pressure-void ratio diagram:

4. Construct the geologic rebound as follows: Estimate the e_x value and then determine the $\left(\frac{e_r + 1}{e_x + 1}\right)$ ratio. C_x/C_r can then be estimated from Fig. 16. The slope of the geologic rebound can then be taken as 0.87 C_x. Construct this geologic rebound through point E (or lower stress on e_o line if sample swelled) and intersecting line x-x at the preconsolidation point P.

6. Reference to Figure 17 shows that the C_x lines, drawn tangent to the average recompression slope, closely pass through the preconsolidation points "P". This phenomena was also noted for most of the other cycled tests. Thus, a line with a slope of C_x (determined in step 4), constructed through point P, may be useful as an upper tangent guide line for the reconstruction of the field recompression curve.

If there is no laboratory test evidence to the contrary, the above procedures are recommended as the best empirical relationships presently available for reconstructing the recompression portion of field consolidation behavior.

Mr. Crisp believes that the recompression curve approximates the slope of the rebound curve only in instances where the rebound curve is very flat. The rebounds from the test results plotted on Figure 6 are quite steep, yet the distinct recompression slopes closely approximate them. Figure 6(A) is presented as part of this closure to broaden the scope of the data on Fig. 6. According to the data presented on his Figure 1, Mr. Crisp's opinion may be based on comparing the final rebound, which starts from the virgin slope, with rebounds starting from the recompression slope. Since virgin consolidation is believed to be associated with a breakdown of the clay structure, the two rebound phenomena may not be directly comparable. In this connection it is interesting to note that the upper test on Fig. 1, which Mr. Crisp reported to the writer to have overburden and preconsolidation pressures of 2.5 and 6 tons/sqft., has an overall recompression slope (upper void ratio points) which is somewhat steeper than the final rebound slope.

As may be seen on Figure 9, the reconstructed slope of the upper portion of the virgin curve, drawn by connecting points P and i, is not sensitive to the position of point P. A large error in estimating the field rebound slope has only a relatively slight influence on the reconstructed virgin slope.

The writer wishes to thank Mr. Crisp for his thoughtful criticisms, and emphasizing the need to present this more comprehensive review of the laboratory data on comparisons of rebound and recompression slopes.

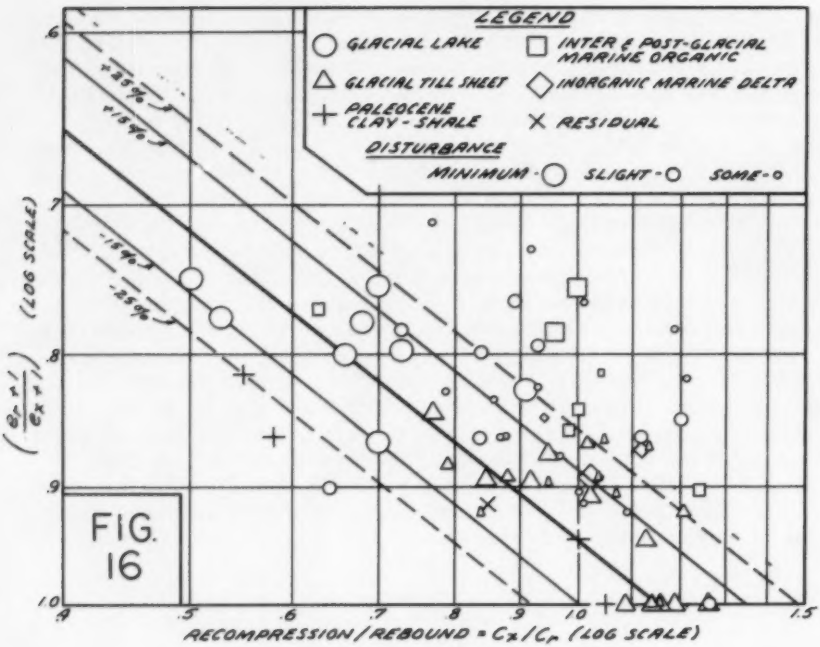

LEGEND
GLACIAL LAKE
INTER & POST-GLACIAL MARINE ORGANIC
GLACIAL TILL SHEET
INORGANIC MARINE DELTA
PALEOCENE CLAY-SHALE
RESIDUAL
DISTURBANCE
MINIMUM
SLIGHT
SOME
+25%
+15%
-15%
-25%
(LOG SCALE)
RECOMPRESSION / REBOUND = Cx / Cr (LOG SCALE)

FIG. 16

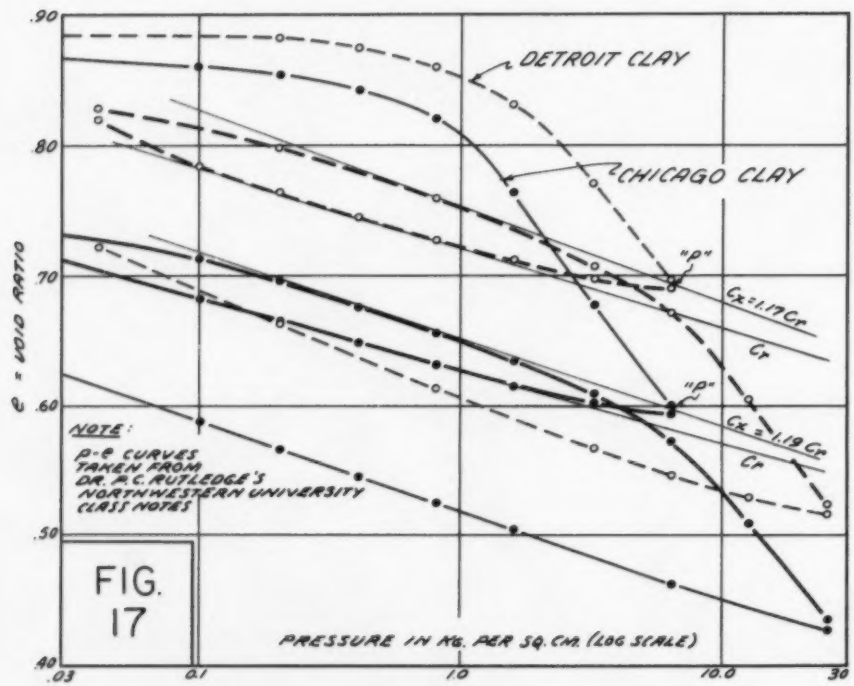
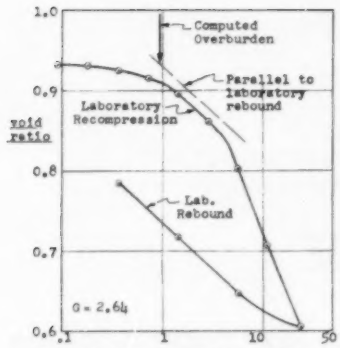
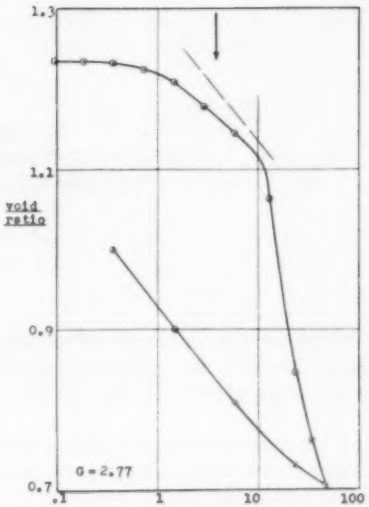

DETROIT CLAY
CHICAGO CLAY
"P"
"P"
Cx = 1.17 Cr
Cr
Cx = 1.19 Cr
Cr
e = VOID RATIO
NOTE:
P-e CURVES TAKEN FROM DR. A.C. RUTLEDGE'S NORTHWESTERN UNIVERSITY CLASS NOTES
PRESSURE IN KG. PER SQ. CM. (LOG SCALE)

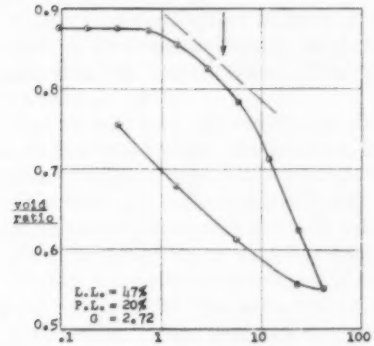
FIG. 17



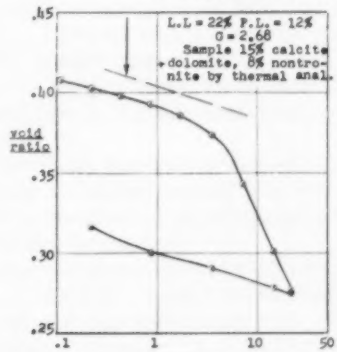
(e) CANADA CLAY (Saskatchewan)



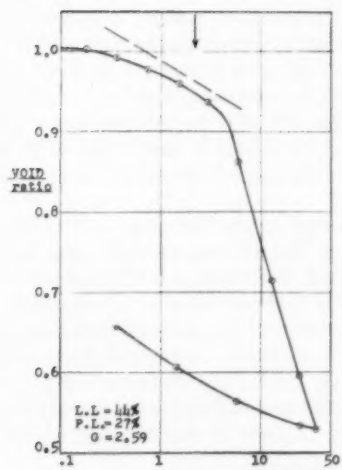
(f) ALBANY CLAY



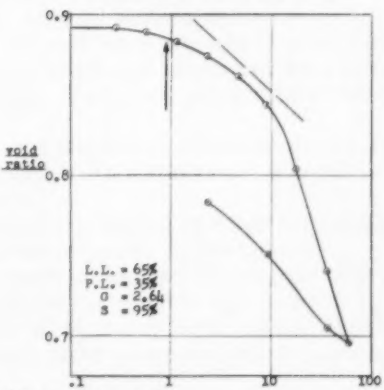
(g) OHIO CLAY



(h) NORTHERN MICHIGAN SILTY CLAY



(i) BRAZIL ORGANIC SILTY CLAY



(j) RESIDUAL DUTCH GUIANA CLAY

FIG. 6(a) - ADDITIONAL SLOPE COMPARISONS

Discussion of
"FOUNDATION CONDITIONS IN THE
CUYAHOGA RIVER VALLEY"

by Ralph B. Peck
(Proc. Sep. 513)

EDWARD C. BAKER,[1] A.M. ASCE—The author's paper is a valuable report on foundation conditions in the Cuyahoga River Valley. As stated, the general condition is that of a compressible, permeable deposit overlying a bed of preconsolidated clay.

Under this condition and for some structures, consolidation of the compressible, permeable deposit by preloading may give a more economical design than the use of piles. This was the writer's experience on a structure completed in 1953.

The structure to be supported was a reinforced concrete silo 120 ft high and 40 ft in diameter. Its location in Fig. 1 is just within the bend of the Cuyahoga River immediately south of the Public Square.

This silo has a capacity of 3,500 tons of sugar. Settlement—particularly unequal settlement—was undesirable, and Mr. A. Casagrande[2] was consulted on the practicability of eliminating piling.

On Mr. Casagrande's advice, a concrete mat foundation was used, but before constructing the silo, the compressible upper deposit was consolidated by preloading. This design and procedure resulted in an estimated saving of $35,000, as compared with the alternate pile foundation.

The excavation for the foundation measured approximately 80 feet square by 10 feet deep. The bottom 5 feet was filled with crushed stone compacted in layers of 6 inches each and choked with the excavated material.

There was encountered in the excavation a group of about 50 old wood piles located along one edge of the foundation. Since these piles might have afforded rigidity to one side of the pad, causing unequal settlement, they were removed rather than cut off. It was interesting to note that the piles at one time supported a coal loading trestle which was abandoned due to progressive settlement of these friction piles which were not driven to the clay level.

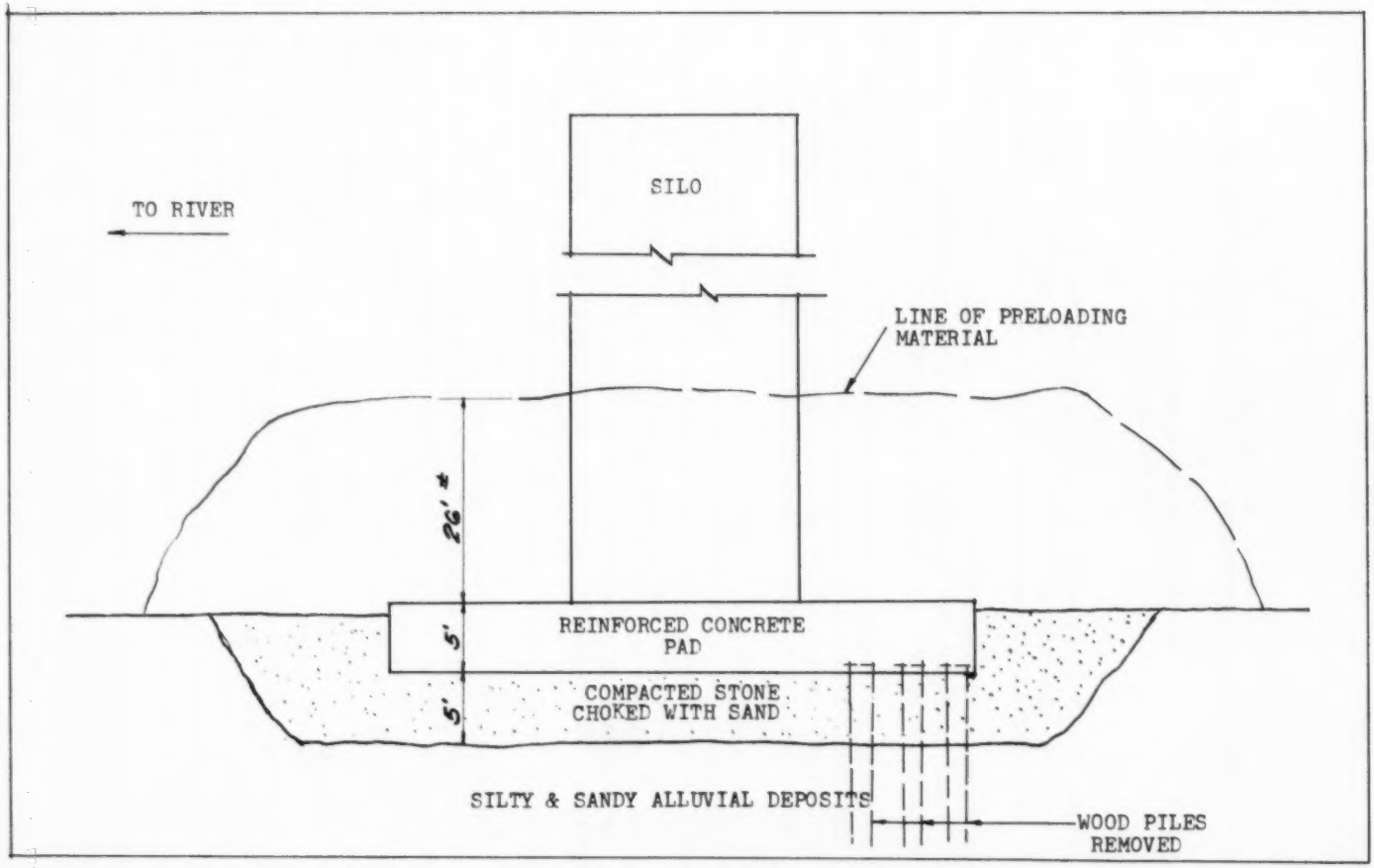
After the stone fill and compaction was completed, the reinforced concrete mat, 65 ft square by 5 ft deep, was formed and poured. The mat was reinforced with two layers of used rails.

When the concrete reached sufficient strength, the mat was preloaded. Because the building site required considerable fill, it was decided to use a dense sand and gravel mixture which would serve the dual purpose of preloading material and site fill. As shown on the writer's figure, the preloading material was piled to approximately 26 ft, which produced soil pressures about 40% in excess of design. Readings taken progressively throughout the

1. Engr., The Austin Co., Engrs. and Builders, Cleveland, Ohio.
2. Gordon McKay, Professor of Soil Mechanics and Foundation Engineering, Harvard University.

preloading operation indicated a uniform settlement of the foundation, with an average of about 3 inches under full preload.

When the preloading material was removed, a rebound of from 12% to 15% of full settlement was recorded. Professor Casagrande, in his original estimate, predicted a settlement of from 2 to 5 inches with a rebound of 10%. Subsequent readings, taken since the silo was completed, filled with sugar, emptied and refilled several times, showed small—and progressively smaller—settlements and rebounds, indicating very satisfactory preconsolidation of the sandy alluvial deposits.



BAKER ON PECK, Fig. 1. - SECTION THRU' SILO FOUNDATION

Discussion of
"ROCK-FILL DAM DESIGN AND CONSTRUCTION PROBLEMS"

by D. J. Bleifuss and James P. Hawke
(Proc. Sep. 514)

I. C. STEELE,[1] M. ASCE—Mr. Bleifuss and Mr. Hawke are to be complimented on the excellency of their paper. There should be many more papers of a similar nature upon the subject of the Design and Construction of Rockfill Dams of all types.

The authors have confined their comments to the composite structure consisting of an impervious zone of earthen materials with rockfill or gravel supporting sections. This composite structure is of two types, the inclined core and the vertical central core.

The writer's experience on rockfills, until recent years, has been chiefly with what he chooses to call the true rockfill type of dam with an upstream face membrane of concrete, timber or gunite. Pacific Gas and Electric Company, with which he was associated for many years, has 14 such dams on its system.

The two composite types and the true rockfill type, when properly designed and constructed, make splendid and durable dams. Selection of the type for a specific site should be determined by site conditions such as character of foundations, available materials, economics and other factors. In some locations, such as in the higher reaches of the Sierra Nevada Mountains in California, impervious material either is non-existent in quantity or too costly to obtain. At such sites the impervious face type of rockfill has proven considerably more economic than concrete dams.

The authors have ably described design and construction requirements for inclined and vertical core dams.

Settlement will occur in both types during construction and when reservoir pressures are applied. There will be significant lateral settlement, from the abutments toward the center of a rockfill, as well as the better known vertical and horizontal movements. Lateral settlement and differential settlements due to abrupt topography changes require careful consideration in order to avoid or minimize cracks. For these reasons, the rockfill should be constructed to give minimum settlements throughout the dam rather than to reduce maximum settlement only, in the deepest sections of rockfill.

Perhaps the areas most vulnerable to the formation of open cracks are those in the vicinity of the high water line of the reservoir and the abutments of the dam where water pressures and rock loads are relatively light and lateral settlement is greatest. Under unfavorable and unfortunate circumstances which usually can be avoided, open cracks might develop in these regions.

Assuming identical characteristics of fill materials and conditions insofar as they are applicable to either type of dam at a given site, it would seem that movement of the fill materials caused by differential settlements would be

1. Cons. Engr., Piedmont, Calif.

more severe in the inclined core than they would be in a central core. Therefore, in constructing the inclined core type of dam greater care must be taken in order to obtain a dependable cutoff and downstream transition zone as well as good compaction in the Main Rockfill section. However, if the dam is well designed and constructed and the impervious zone is completely confined between the two transition sections as it is intended to be, it is the writer's opinion that the pressures exerted upon and through the moist inclined core material are sufficient to prevent formation of open cracks during the process of more or less gradual settlement of the underlying fill materials.

The writer agrees that rockfills, particularly of large size stones, can withstand considerable flow through the fill and over pour but such should not be considered in the spillway design. However, economies in diversion and construction may be attained by taking a calculated risk of overpour during construction. In the case of Dix Dam, mentioned by the authors, flood waters rose rapidly during the construction period and submerged about 37,000 sq. ft. of uncovered placed rock and dumped fill. The difference between reservoir level and tailwater was about 36 feet. A very large volume of water passed through the dumped rock fill with no noticeable displacement of rock.

Pacific Gas and Electric Company's Upper Bear River Dam withstood, for several hours, overtopping to a maximum depth of 15 inches. The dam is 75 feet high and has an upstream face membrane on slopes of 1/2 and 3/4 to 1. The downstream face consisted of a single layer of well placed dry rubble laid on slopes of 1/2, 3/4 and 1 to 1. The fill between the upstream and downstream rubble faces consisted largely of a mixture of small rock and fines. After the flood it appeared that there had been some movement of the downstream rock facing. Subsequently the dam was reinforced by dumping rock against the downstream face and the spillway was enlarged. A currently constructed rockfill of large, well-sluiced rock would certainly withstand such overpour with a greater margin of safety.

The authors state that rockfill usually has a shear coefficient greater than 0.7. In the writer's opinion, dumped and sluiced quarried rockfill becomes so interlocked that a shear coefficient of unity can conservatively be used in design. The existence of many dams with 1/2:1, 3/4:1 and 1:1 slopes and nominal amounts of surface placed rock is substantial evidence of a high shear value.

The writer is in full accord with the statement that reasonably clean and compact gravel may be used as a foundation except under the impervious zone. Careful examination of several river bed gravel deposits has revealed essentially complete rock to rock contact through sizes ranging from large boulders to coarse sand with interstices completely filled with smaller rocks and sand particles. Such deposits surely are more compact and secure than any artificial fill that can be made to replace them.

The authors mention natural dumped rockfill slopes ranging from 1.35 to 1.40 horizontal to 1.0 vertical. The writer's experience, generally, has been with slopes of 1.30 to 1.35 horizontal to 1.0 vertical.

The 172-ft. Cogoti Dam in Chile was built to an average downstream slope of 1.6 to 1. It withstood an extremely severe earthquake with practically insignificant disturbance of the fill. The vertical settlement caused by the quake measured 1.4 feet.

The author's comments in regard to quarry operation, dumping of rock, and the amount of permissible fines are especially commendable.

The problem of permissible fines is difficult. In writing Specifications, percentages seem to be the only means of specific control. In the field it becomes a matter of judgment, and differences of opinion between constructor and owner arise. The writer has found that a pricing schedule that causes both the constructor and the owner to lose some money on rejection of material does much to solve the problem.

The authors' statements favoring high lifts and heavy sluicing of the rockfill are well taken. Assuming suitable rock is available, these two factors are most important. High lifts are more adaptable to the inclined core type than to the vertical core type. This is a distinct advantage construction-wise and will give the minimum settlement in the main rockfill.

The behavior of the 275-ft. San Gabriel Dam No. 2, a rockfill with upstream membrane, is cited. The dam was constructed in 25-ft. lifts and sluicing was prohibited. The rock, though hard, contained some weak cleavage planes. Measurements taken during the later stages of construction indicated an accumulated total settlement near the crest of more than 15 feet in seven months. Of this amount, the last several feet of settlement came in one week of heavy rainstorms during which 12 inches of rain fell in one day. One effect of that settlement is shown in Fig. 1. An extensive, deep crack 9 inches to 14 inches wide developed near the right abutment and the crest of the dam. This is in contrast to a maximum crest settlement after two reservoir fillings of 1.13 feet (0.5 % of the 240 ft. height) of Pacific Gas and Electric Company's new Bear River concrete face rockfill dam, constructed in high lifts and well sluiced.

The La Joie Storage Dam, British Columbia Electric Company, Ltd., on Bridge River is of the rockfill type with a temporary timber face overlying a 14-inch unreinforced concrete sub-slab. The deepest fill overlies the river channel at old La Joie Falls where the foundation and abutment slopes are quite steep. Here the alignment of the dam is concave to the reservoir. The dam was constructed originally to a height of about 215 feet and presently is being enlarged to its ultimate size. Differential settlement near the right abutment of the original structure caused the formation of an extensive crack in one of the timber sills. At the point of maximum separation the crack measured about 7 inches in width and there was a 3-inch downstream offset. Adequate repairs were made and little further settlement occurred.

The writer agrees with the authors' emphasis on sluicing but not entirely with the methods described by them. Fortunately, it appears that the quantity of water applied at the time and location of dumping is more important than the method of applying the water. Usually the fines tend to accumulate at the point of dump and for a short distance down the slope. Concentration of large volumes of water in these areas is more effective than high nozzle velocities. Also, there is beneficial action beneath the surface by erosion from occasional steady flow at one point. However, one should continually strive to apply water in the most effective manner.

Excellent results also have been obtained in many cases with the monitor located on the rockfill bench to one side and a short distance back from the point of dump as illustrated in Fig. 2. The stream is played diagonally into the rock as it slides off the truck and on down the rockfill face. The monitors are mounted on short skid-type carriers with a liberal length of hose. They can be speedily moved to new positions by the bulldozer which normally is in use on the rockfill bench. A long pipe handle fastened to the monitor barrel, and counter-weighted, permits easy operation of the monitor by one man.

In conclusion, the writer desires to state that he is in agreement with the authors that a certain amount of fines, with sluicing, acts as a lubricant. Slides frequently occur where water is applied in quantity, especially to areas containing local concentrations of fines, and do permit better adjustment of individual rocks. However, he prefers a minimum amount of fines and would not advocate deliberate inclusion of fines in order to accommodate such action.

Fig. 1 - View showing 14-inch crack in rockfill during construction, San Gabriel Dam No. 2.

Fig. 2 - Sluicing of dumped rockfill with skid mounted, hand operated monitor during construction of Pacific Gas and Electric Company's lower Bear River Dams.

Discussion of
"ENGINEERING PROPERTIES OF EXPANSIVE CLAYS"

by W. G. Holtz and H. J. Gibbs
(Proc. Sep. 516)

WILLIAM T. ALTMEYER,[1] A.M. ASCE—The approach of the authors to the identification of expansive clays, by coincidence, closely parallels experiments performed by the writer and other engineers of the Donald R. Warren Co. Several of the authors' conclusions are substantiated by the results obtained in our tests and it is the intent of the writer to supplement Mr. Holtz and Mr. Gibbs' article with identification criteria he has found to be equally significant and valuable.

In 1951 the tremendous increase in the construction of new homes in the Los Angeles area inevitably had led to the development of marginal-type land where deep deposits of both residual and transported clays, known by the colloquial term "adobe," represented the foundation medium. The Federal loan guaranty agencies were besieged with complaints by the Veteran-owners of heaving foundations and galloping porches. Recommendations on establishing criteria for the identification of clay soils whose volume change with moisture variations was critical were urgently requested.

As Mr. Holtz and Mr. Gibbs pointed out in their article (Page 1), microscopic examination, X-Ray diffraction analysis and differential thermal analysis are tests useful in determining the amount and type of mineral present, particularly the highly expansive montmorillonite. These tests were rejected by the writer and his associates as being too time-consuming and requiring special expensive equipment and highly skilled laboratory techniques. They were of the opinion that the commonly used Atterberg limits could provide reliable indicators of the degree of expansion to be expected for a given clay soil.

The criteria developed by the Donald R. Warren Co. includes the shrinkage limit although within more narrow boundaries than those selected by the authors to represent indications of low, medium, high and very high expansion. The range of shrinkage limits in the medium range (8-18) established by the authors (Page 5), overlaps their very high, high and low ranges. The criteria adopted by the Donald R. Warren Co. with respect to anticipated volume change as indicated by the shrinkage limit are as follows:

Shrinkage Limit (%)	Degree of Vol. Change
<10	Critical
10-12	Marginal
>12	Non-Critical

The authors gave a shrinkage limit of greater than 13 as indicating a low degree of expansion (Page 5). A very high degree of expansion was indicated by a shrinkage limit of less than 10 which, by coincidence, the writer and his

1. Foundation Engr., Donald R. Warren Co., Cons. Engrs., Los Angeles, Calif.

associates selected as indicative of a critical degree of volume change.

The percent of colloids used by the authors was rejected for two reasons: first, the classification system used by the writer's company (The Corps of Engineers Uniform Soil Classification) does not depend on hydrometer analyses for its descriptions and the percent of colloids would require additional testing in the laboratory for questionable utility as evidenced by the wide overlap of values reported by the authors; secondly, the percent of colloids is relatively meaningless unless it is known also whether the colloids represent minerals of the montmorillonite, kaolinite or illite groups. The writer and his associates feel that other indices are more conclusive.

Considerable surprise was felt in reading the article that no employment of the percent of lineal shrinkage was made. The authors mentioned (Page 5) that the plasticity index (PI) demonstrates the range of moisture content possible in the plastic range and is, consequently, indicative of volume change to be expected. While there is no doubt that the above relationship holds, the writer and his colleagues believe that the percent of lineal shrinkage occurring when the moisture of a soil mass is reduced from an upper limit, commonly taken to be the field moisture equivalent, to the lower limit below which no volume change occurs—the shrinkage limit—is more meaningful. This conclusion is based on the fact that the soil in place will tend to shed water before the liquid limit is reached. The field moisture equivalent, consequently, represents a more realistic upper limit.

The following boundaries were established with respect to percent of lineal shrinkage:

Lineal Shrinkage (%)	Degree of Vol. Change
>8	Critical
5-8	Marginal
<5	Non-Critical

It should be pointed out that the boundaries established were based on the results of a large number of laboratory tests performed on soils encountered in the greater Los Angeles area and, as is the case with any set of arbitrary limits, borderline conditions and even contradictions will occasionally result. As frequently occurs in the practice of soil mechanics, the indices are relative rather than absolute.

Additional confirmation of the degree of volume change to be anticipated was sought in the form of actual expansion under loads typical of residential construction. Remolded specimens of the soil under test are allowed to saturate under a normal load of 650 lbs. per sq. ft. This load is roughly equivalent to footing dead loads for single-story wood frame structures. As pointed out by the authors (Fig. 9), the amount of expansion is greatly dependent on molding moistures and densities. The writer's company decided to test two specimens. One specimen is loaded at approximately 85 percent of maximum density and a moisture content equal to the shrinkage limit, and then allowed to saturate under the above mentioned restraining load. This test condition, it was felt, represented conditions in undisturbed soil. Since many homes in the Los Angeles area are built all or in part on compacted fill, a second expansion test is performed on a specimen remolded to 90 percent of maximum density and at optimum moisture content. Again the specimen is allowed to saturate under a normal load of 650 lbs. per sq. ft. This test represents the behavior of compacted fill under footings. The more critical of the two cases is then considered to govern with respect to the

following ranges:

Percent Expansion	Degree of Vol. Change
>1.5	Critical
0.5-1.5	Marginal
<0.5	Non-Critical

The expansion test is performed in a standard fixed-ring type consolidometer. A remolded specimen approximately $2\frac{1}{2}$ inches in diameter and 1 inch in height is loaded by means of a beam. The vertical expansion during saturation is measured by a dial indicator.

The value adopted for critical expansion (1.5 percent) is considerably smaller than that presented by the authors for very high expansion (30 percent) because of the difference in the restraining loads used in the expansion tests (650 p.s.f. as compared to 144 p.s.f. or 1.0 p.s.i.). The recommendation is frequently made where the clay soil tested is in the marginal range that special precautions be taken with lightly loaded flatwork such as porch and garage slabs, patios, etc. These precautions would include use of wire reinforcing mesh, non-expansive backfill under raised porches and casting garage slabs separate from the footings.

Considerable reduction in the amount of anticipated expansion can be obtained by avoidance of overcompaction of expansive soils and placement at moisture contents as high as practical for good operation of construction equipment. Engineers of this organization have recommended ponding and saturating clay soils on which slabs will be cast to raise the in-place moistures to the upper limit of the moisture range that produces volume change. Once placed, capillary attraction and presence of the slab will keep seasonal changes of moisture to a minimum and consequently, volume change of the foundation soil will be negligible.

CONCLUSIONS

Commonly used soil laboratory tests such as the shrinkage limit and percent of lineal shrinkage can be employed as qualitative indices of expansiveness of clay soils. A quantitative measure of potential expansion can be obtained by saturating remolded specimens of soil in the consolidometer under restraining loads equivalent to anticipated footing loads.

Considerable variations in the amount of expansion can be produced by molding moisture, density and sample preparation methods. Further studies in standardizing the test procedures are being conducted by the writer in connection with a research program leading to graduate degree from the School of Engineering at the University of Southern California.

EDWARD S. BARBER,[2] A.M. ASCE—The properties of expansive clays are well illustrated in this paper. The following data supplement, and, in general, agree with the authors' presentation.

Table A, like Figure 5, shows the expansion or swell of various compacted soils when immersed and unloaded gradually as compared to the swell when immersed under the final load of 20 pounds per square foot. Table B shows the effect of various loads at the time of immersion for the A-6 clay. The

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first two columns correspond to the assumed curve in Figure 6.

The effect of temporary restraint (movement prevented) decreases as the moisture at the time of compaction increases, as shown in Table C. This is because higher moisture contents produce more uniform arrangement of the soil particles (structure) which reduces the opportunity for internal rearrangement of particles when the immersed soil is restrained from swelling.

Table D shows the swell of duplicate samples immersed under a heavy load and then unloaded after various times. While the swell from initial thickness reduces with loading time, the swell from the time of load release is constant, indicating that the reduction of swell by temporary restraint is due to reorientation of particles without a release of the elasticity stored in their compression and bending.

The effect of structure is also shown by different methods of compaction. In a typical case, tamping, which gives a more uniform structure, gave one-third less swell than static compaction for the same soil at the same moisture and density.

Figure A, showing the swell under a load of 350 pounds per square foot after molding to various moistures and densities, is similar to Figure 8. For the combination of low density and low moisture content in Figure A, the swell is less for the very dry soil because the initial soil contained hard lumps of clay as large as the number 10 sieve size with a relatively uniform internal structure which predominates over the effect of the static load used in forming the samples.

In Table E is shown the reduction of swelling pressure with increase in permitted displacement. The first value is comparable to those shown in Figure 8. In Table F, similar effects of variations in moisture content and density on swelling pressure are shown as well as the effect of the time allowed for rebound of the sample before setting it against the restraint. If sufficient time is not allowed for the rebound after compacting and before the restraint is applied, an increased pressure is recorded. The low reaction (50 pounds) for the 20-percent moisture content at 118 pounds per cubic foot is due to the fact that water was squeezed from the sample into the mold during molding and reabsorbed during the time allowed for rebound.

Table G shows the increase of swelling pressure and swell with increase of compacting load. The maximum swelling pressure shown, 4520 pounds, is 50 kips per square foot or 360 pounds per square inch.

Although lacking the arid climate contributing to several of the authors' field situations, important movements have been observed involving a marine clay (Tuxedo) in the coastal plain near the District of Columbia. The writer has investigated an apartment house which was badly cracked when a stairwell was lifted an inch by swelling clay. Several other structures have cracked due to drying of the same clay, especially when aided by the roots of trees. Figure B shows a wall distorted by the shrinkage craters over the root systems of a row of trees. Figure C shows similar settlement of an iron fence.

Table A.--Maximum swell for different soils under different loads [1/].

Soil	Cumulative Thickness increase under various loads in kips per sq. ft.							
	8	4 [2/]	2 [2/]	1 [2/]	0.5 [2/]	0.2 [2/]	0.02 [2/]	0.02
	Percent	Percent	Percent	Percent	Percent	Percent	Percent	Percent
A-1	-0.4	-0.3	-0.1	0.0	0.1	0.5	2.1	11.8
A-2	0.0	0.6	1.0	1.4	2.0	2.7	6.6	13.1
A-3	0.0	0.0	0.2	0.2	0.3	0.3	0.6	0.4
A-4	0.8	1.9	3.2	4.3	5.7	7.4	12.9	22.5
A-5	0.5	2.1	2.9	4.6	6.2	7.8	15.1	17.1
A-6	12.2	13.9	16.0	18.2	23.1	-	38.4	70.7
A-7	1.4	4.7	5.5	6.6	7.6	9.0	13.2	21.4
A-8	10.1	12.9	15.6	16.4	17.4	19.1	63.5	98.7

1/ Air-dry soil compacted under 500 kips per sq. ft.

2/ After swelling under previous load.

Table B.--Effect of load at time of immersion on swell[1/].

Load at time of immersion	Maximum thickness increase	
	Under immersion load	Under 20 pounds per square foot
Lb./sq. ft.	Percent	Percent
8000	12.2	38.4
4000	16.1	33.8
2000	19.3	34.0
1000	27.7	42.7
500	30.1	43.5
200	40.5	51.8
20	70.7	70.7

1/ Air-dry A-6 soil compacted under 500 kips per sq. ft.

Table C.--Effect of temporary restraint on swelling [1/].

Initial moisture content	Thickness increase with sorption of water when	
	Restrained 24 hr.	Unrestrained
Percent	Percent	Percent
15	7.3	12.3
18	6.4	12.3
21	6.2	8.9
24	5.5	4.9

1/ Tuxedo clay mixed to various moisture contents and compacted to 103 p.c.f., allowed to sorb water under 170 p.s.f.

Table D.--Swell after sorption of water under compacting load 1/.

Time between water application and load release	Thickness increase under 20 p.s.f.	
	From initial thickness	From thickness at time of load release
	Percent	Percent
1 minute	17	17
1 hour	7	17
1 day	4	15
1 week	2	16
1 month	1	17
1 year	2	19

1/ Air-dry Tuxedo clay loaded to 16 kips per sq. ft., water applied while loaded, load reduced to 20 p.s.f. after various times.

Table E.--Effect of strain on swelling pressure [1/].

Displacement permitted		Maximum pressure developed by sorption
Inches	Percent of thickness	Pounds
0.000	0.0	2520
0.006	0.6	1900
0.012	1.2	1500
0.018	1.8	1220
0.024	2.4	950

1/ Tuxedo clay compacted to 118 p.c.f. at 10 percent moisture, 1-inch thickness, 4-inch diameter

Table F.--Swelling pressure against fixed restraint 1/

Moisture content	Swelling pressure for given density and time between compression and setting of restraint		
	118 p.c.f., 5 min.	118 p.c.f., 24 hr.	107 p.c.f., 24 hr.
Percent	Pounds	Pounds	Pounds
0	2970	2280	640
10	2510	2080	610
15	2120	660	370
20	1070	50	210

1/ Tuxedo clay staticly compressed, 4-inch diameter.

Table G.--Swelling pressure and swell of two soils.

Compacting load on 4-in. diameter	Swelling pressure		Subsequent swell under 20 p.s.f.	
	Tuxedo clay	Organic silt	Tuxedo clay	Organic silt
Kips	Pounds	Pounds	Percent	Percent
1	20	20	1	3
10	620	250	12	7
20	1280	580	18	11
40	3280	1140	26	14
60	4520	1700	30	18

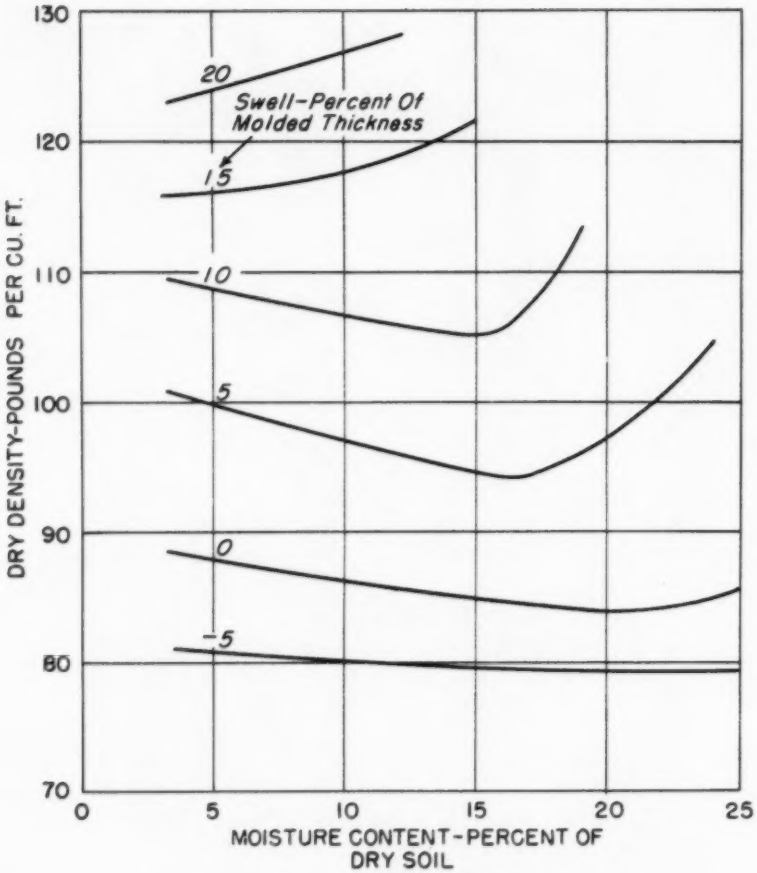

Figure A.--Swell of Tuxedo clay when immersed under 350 pounds per sq. ft. after molding to different moistures and densities

Figure B.--Distortion of wall by row of trees.

Figure C.--Settlement of fence by row of trees.

LAWRENCE A. DuBOSE,[3] A.M. ASCE—After the comprehensive treatment of the expansive clay problem by Messrs. Holtz and Gibbs, the soils engineer should have little difficulty in recognizing soils possessing this undesirable property. While numerous papers have discussed soils from the standpoint of density, moisture content, surcharge, volume change and strength, seldom has an attempt been made to correlate these factors with expansive clays.

The design of compacted fills to meet anticipated future needs is a phase of soils engineering that deserves more emphasis than it has received. In normal fill construction the selection of a moisture content and compaction effort combination to produce a desired strength is a relatively simple procedure. In the Southwest where expansive clays are not uncommon, the swelling under low surcharges complicates the design problem. However, by performing volume change and strength tests for soils compacted at several moisture content and applied energy combinations, curves can be established to permit a rational fill design. One method for illustrating these relationships is shown by the curves in Figures 1 and 2.

These data are from research performed on Taylor Marl Clay, a troublesome Texas soil. The liquid limit for this soil was 75 percent, the plasticity index 53 percent, the shrinkage limit 12 percent and the colloid content 40 percent. The research in this study included volume change and strength tests for specimens compacted to optimum conditions using efforts varying from $\frac{1}{2}$ to 5 times the Standard Compaction.

The left portion of Figure 1 illustrates the increase in optimum density for increased compaction and the right curve gives the strength of specimens compacted to this condition. The figure can easily be extended to show similar relationships for samples compacted to specified percentages on the wet or dry side of the optimum. Strength tests can also be included for anticipated future moisture conditions.

The surcharge pressures required to prevent volume changes for Taylor Marl specimens compacted to optimum conditions are shown in Figure 2. This curve was established in essentially the same manner as described by Holtz and Gibbs using consolidometers. It was found that the use of $\frac{1}{2}$ inch thick rings reduced the testing time by over fifty per cent and gave comparable data to the $1\frac{1}{4}$ inch thick rings.

The curves in Figures 1 and 2 may be used to select a moisture content and compaction effort that would both limit the volume changes and provide the necessary strengths. For high fills it might even be desirable to use several combinations as the surcharge decreases from the lower layers to the top of the fill.

Another interesting relationship found in the Taylor Marl research was the time versus swell curves. The data in Figure 3 show that the selection of some time interval, for example, four days, may result in incorrect volume change conclusions. One notes that the expansion is divided into "primary" and "secondary" swell. For the $\frac{1}{2}$ and $1\frac{1}{4}$ inch thick specimens the primary swell occurred in the first 4 days. For the five inch specimens, only about thirty per cent of the primary swell occurred in the first four days. After specimen number 2 had been permitted to swell for 55 days to give a total volume change of over 20 per cent, moisture determinations were made at 1 inch intervals. The values were 25.1, 27.5, 30.2, 33.5, 35.0 and 38.2 per cent as measured from the bottom to top of the sample. The initial moisture

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content was 15 per cent. While this example represents an extreme case, it does serve to illustrate the tremendous swell potential possible under certain conditions. Such conditions can be avoided for fills, but in natural deposits allowances must either be made to prevent movement or to design the structure to take the movement. When it is practical to design for the latter case, the soils engineer needs a more definite procedure for determining the magnitudes of future heave.

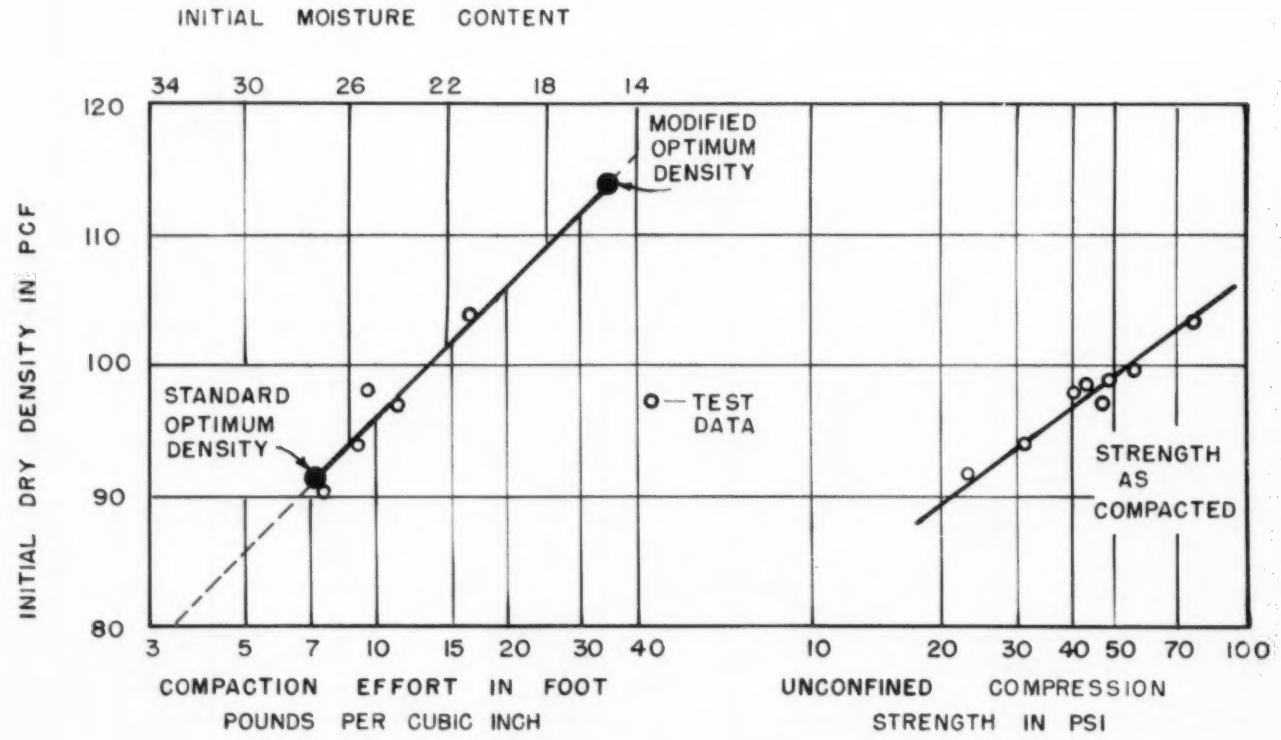

FIGURE 1 — STRENGTH AND DENSITY RELATIONSHIP FOR SPECIMENS COMPACTED AT OPTIMUM CONDITIONS.

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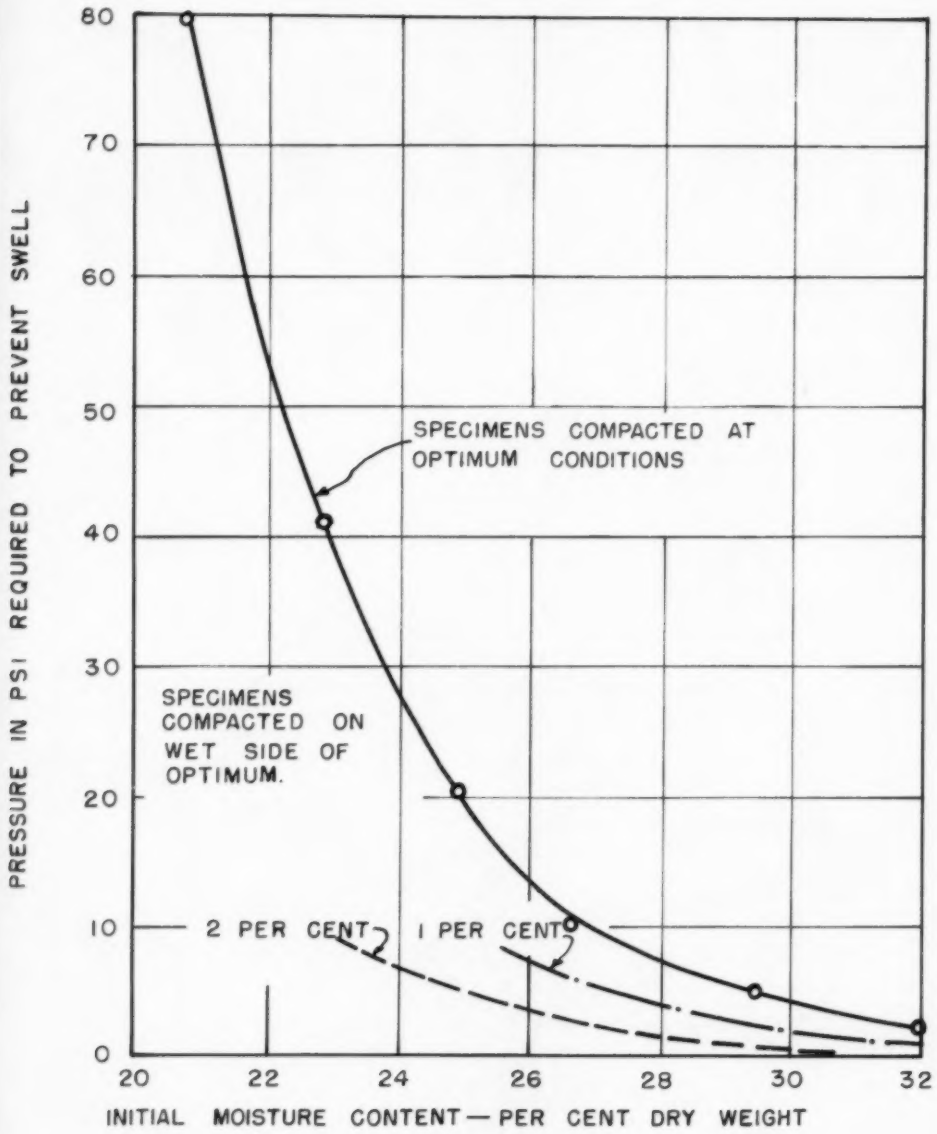

FIGURE 2 — RELATION OF INDICATED FACTORS TO RESULT IN ZERO VOLUME CHANGE.

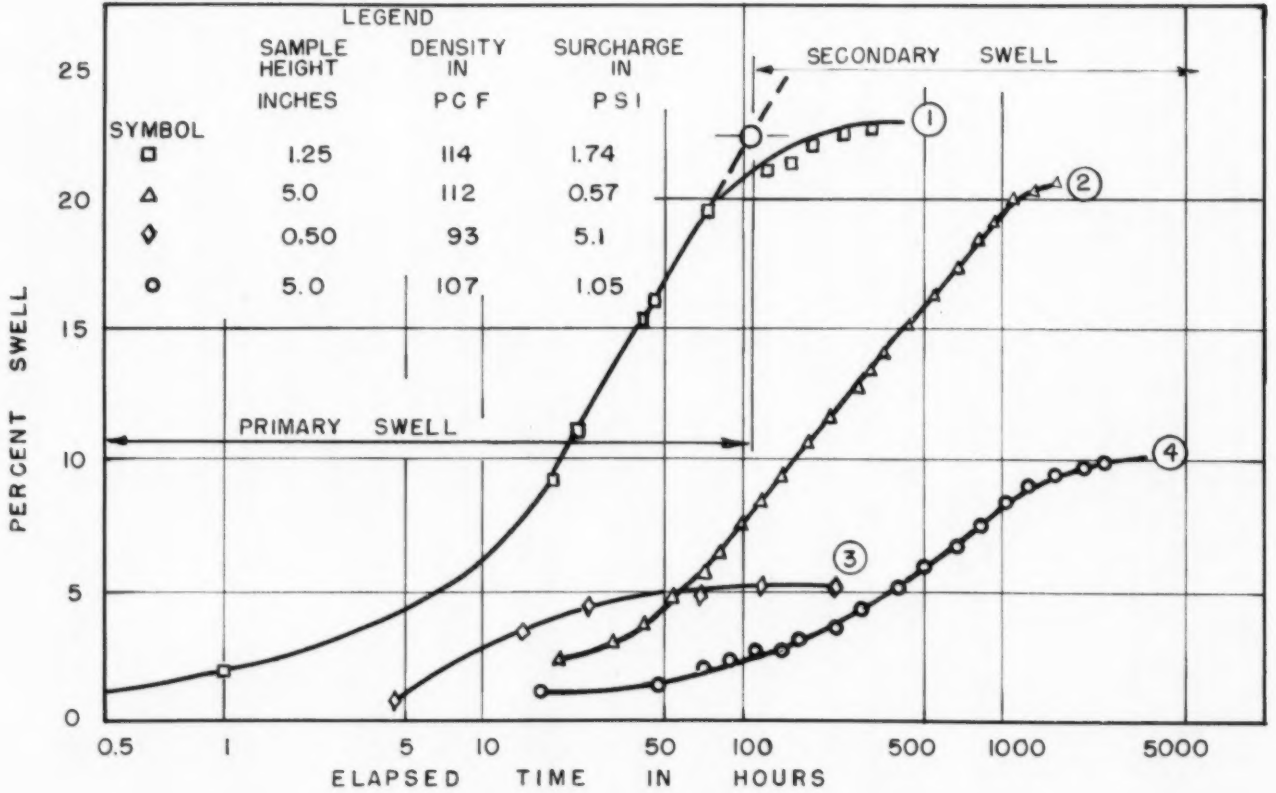

FIGURE 3 —— THE INFLUENCE OF SAMPLE THICKNESS ON "SWELL" TIME.

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PROCEEDINGS-SEPARATES

The technical papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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